# Designing Boston's Storrow Drive Connector Bridge

## Steel Alternative Chosen for Cost and Constructability

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One of the largest steel box girders in North America carries a crucial link in Boston's Central Artery/Tunnel project, the Storrow Drive Bridge. This single-cell trapezoidal steel box girder bridge is designed to support the 76-foot wide roadway connecting Boston's Storrow Drive and Leverett Circle with I-93. The exceptional size and weight of the box girder sections, and the resulting design and constructability considerations, makes the Storrow Drive Bridge an excellent case study of today's modern steel-bridge design and construction.

The Charles River Bridges project involves two major bridge structures over the Charles River: the main-line cable-stayed bridge and the Storrow Drive Bridge. The two new bridges will more than double the existing six lane capacity over the Charles River, eliminating an existing bottle-neck condition and vastly improving the sub-optimal traffic flow pattern that exists today.

The main-line cable stayed bridge will carry a total of ten lanes of traffic, including eight lanes of I-93 traffic and an additional two-lane ramp serving downtown Boston and Sumner tunnel traffic. The Storrow Drive connector will carry a total of four lanes of traffic linking Storrow Drive/Leverett Circle to I-93.

The design of the two new bridges and the demolition of the existing double-deck truss bridge were lumped into one design contract by the CA/T Project and awarded to HNTB. The design documents for the two new structures and the demolition of the existing structure were bid and let separately to suit the Project master schedule. The construction contract for the Storrow Drive Bridge was awarded to the lowest bidder, Daniel O'Connell (DOC) of Holyoke, Mass., for a sum of $22.3 million.

Early in the design process, HNTB contacted numerous steel fabricators for their input. Tampa Steel, which DOC selected as the steel fabricator, was one

of the very few who suggested fabricating the entire 830-foot long box girder in sections and delivering them by ocean-going barge.

Designs for the Storrow Drive Bridge were prepared in both steel and concrete alternates for competitive bidding. Figg Bridge Engineers, acting as a sub-consultant to HNTB, designed the concrete alternate. Both designs conformed to the same general geometric layout in terms of the superstructure cross section, span and piers. The overall bridge layout, defined by CA/T Project, consisted of a 3-span structure measuring 830'-0" in total length with a main span of 380'-0". The superstructure supporting the 76'-0" wide roadway is supported on a variable depth box-girder superstructure 18'-0" deep at the main piers. Visual compatibility with the adjacent main-line cable-stayed bridge also was a key issue.

The HNTB team optimized the designs for both alternates within the owner requirements, gaining project permission to implement many design innovations throughout the course of the final design development.

The bridge also is a testament and a showcase validating the use of such large single-cell steel box girder superstructures. It is an innovative and striking demonstration that a steel box girder of these dimensions can be designed and constructed economically in a competitive marketplace, challenging the stereotypical notion that steel box girders of this magnitude may not be economical nor logistically viable in competition with other types of designs.

The design and construction schedule for the Storrow Drive Bridge has been governed by the early opening of the Interim Leverett Circle Connector planned for this fall. This early opening precedes the rest of openings in the Area North of the Causeway by three-to-five years. Northbound I-93 will open in year 2002 and southbound I-93 will open in 2004.

**Key Design Challenges:** The key design challenges on the Storrow Drive Bridge project were primarily due to the following:

- The exceptional size and the weight of the box girder sections and the resulting design and constructability considerations;
- The weak and highly variable rock conditions in the general area requiring special measures in foundation design;
- The potentially liquefiable soil deposits, especially on the south bank;
- Visual harmony with the abutting structures and the cable stayed bridge.

**Steel Box Girder Design and Detailing:** The main steel box girder measures 34'-6" out-to-out of top flanges. It varies in depth from 8 feet at the end piers to 18 feet at the main piers and to 10 feet at the center of the main span.

The variable depth girder is 31'-0" wide (c/c webs) at the top flange level. The constant web slope and the variable girder depth results in a bottom flange width of 18'-0" at the main piers to a maximum of 25'-0" at the end piers. The 76-foot roadway width is achieved by using floor-beams with cantilever outriggers extending out a typical distance of 17'-6" beyond the box girder. The entire floor beam measures 66'-0" end-to-end, and consists of three sections with one inside the box girder and two outer cantilever sections. The outer ends of the cantilever out riggers also frame into a fascia girder. The fascia girder varies in depth from approximately 1'-8" at the center of the main span to a depth of about five feet at the end piers. The exact depth of the fascia beam at the end piers is set to match the structure depth of the abutting viaducts at the respective interfaces. In addition to its aesthetic function of providing a smooth transition and hiding utility lines from direct view, it also serves to distribute wheel loads in the longitudinal direction, optimizing the design of the cantilever outriggers.

The use of a single box girder with floor-beam cantilevers was chosen due to aesthetic considerations. The feasibility and the economy of using higher strength steels for the top flange and the use of a steel-concrete composite bottom flange at the main pier locations also were investigated at the onset of the final design, but were ruled out.

During the early phases, a number of fabricators and erectors were contacted and consulted to obtain input on the proposed details, size of the fabricated girder sections, shop assembly, lifting and feasibility of transportation to the site. This fabricator survey indicated that while most had to rely entirely on over-

land transportation, a few could ship sections by barge. Barge size restrictions through the Charles River locks also were considered when confirming the feasibility of water transportation.

In the final design and detailing, optional longitudinal splices were provided to comply with size restrictions required for truck transportation and to facilitate erection and handling as needed.

The normal field splices were provided to sector the 830-foot girder into nine separate field sections. The optional longitudinal splices in turn sub-divided these field sections. The 350-ton main pier segments, deepest and heaviest of all, were provided with a total of three longitudinal splices. Two splices were located halfway down on each web and a third at the center of the 18-foot wide bottom flange, separating it into four sections. For the end sections, the height was not a problem. The 26-foot wide bottom flange, however, was provided with two longitudinal splices at third points.

The superstructure required 1860 tons of structural steel. Over the main piers, the box section consists of 4" x 54" top flanges, 2" thick bottom flanges and 1-1/4" web plates. Over the negative moment regions near the main piers, the box girder bottom flange was stiffened with WT 16.5 x 100.5 welded in the transverse direction and six lines of WT 10.5 x 36.5 in the longitudinal direction, longitudinal stiffeners detailed to pass through web openings of the transverse stiffeners. Over the positive moment areas, the longitudinal stiffeners were changed to 3/4" x 8" plates with MC 6 x 11.5 used in the transverse direction connected to the longitudinal stiffeners at the top with clip angles. The box girder field splices were made with ASTM A325 1" diameter bolts. All other bolted connections used 7/8" diameter bolts, including the optional longitudinal splices. The two fixed pot bearings atop each of the main piers have a capacity of nearly 5900 kips each. The expansion pot bearings at the end piers are nearly 1000 kips each.

The Storrow Drive Bridge is abutted by a curved steel multi-box girder viaduct at the south interface and a concrete segmental viaduct at the north interface. Modular type expansion joints were used between the Storrow Drive Bridge and the abutting viaducts at both interfaces.

**Substructure Design:** The single column main piers taper from a 20'-0" width at the top down to 16'-0" at the base in the transverse direction. In the longitudinal direction, the shaft width is 9'-6" at the top, increasing at a uniform taper with decreasing elevations. The main piers are founded on 11'-0" thick 36' x 36' footings supported on four 8' diameter drilled shafts. The frame type end piers consist of a cap beam supported by two circular pier columns. The 7'-3" diameter pier columns are supported by 8' diameter single drilled shafts concentric with the column.

Examination of the rock-core data from geotechnical investigations available during design indicated that the rock quality was highly variable. The rock profile in the area consisted primarily of intermixed layers of rock deposits of Silty Argillite and Clayey Argillite, designated SC-1, SC-2 and SC-3. This is a light-gray, soft-to-medium-hard rock type. Haley & Aldrich, Inc., geotechnical subconsultant to HNTB, provided foundation recommendations.

The design of 8-foot diameter drilled shaft rock sockets was based on an expected thickness of different layers established with available subsurface explorations during design. During construction, the contractor was required to obtain additional rock core samples and submit for engineering evaluation an appropriate adjustment of rock socket length. The working loads for drilled shafts at the main piers were 4500kips each.

**Seismic Considerations:** The soil conditions indicated the presence of liquefiable layers at the riverbanks near end piers. As an alternate to the soil remediation suggested in the Project Design Criteria, HNTB proposed and investigated the possibility of designing the drilled shafts against the forces developed by lateral spreading of the post-liquefied soils. This approach was found to be both feasible and cost effective in comparison to other potential solutions. At most locations, the liquefaction considerations did not govern the design of the drilled shafts. At the south end pier where this problem was more significant, a 1-1/2" permanent steel outer casing and additional reinforcing steel was provided in the drilled shafts.

**Bridge Fabrication and Steel Erection:** After con-

sidering the availability and feasibility of lifting equipment, DOC planned to lift the field sections using a barge-mounted Manotowac 4100 ringer crane and a land-based M250. The access limitations for picking and placing did not permit the lifting of the 350-ton pier segments in whole. Thus, for these main pier field sections, only the longitudinal web splices were used to sector the main pier field section into a lower section and an upper section. Some of the sections were lifted in tandem by the two cranes.

The 830-foot box girder was shop assembled from end-to-end, including the cantilever out riggers and the fascia girders, and surveyed to verify the cambered geometry. After the shop fit-up, they were disasssembled and shipped to Boston via ocean-going barges.

First the south and north main pier segments were erected on permanent piers and stabilized by false work towers. The lower half was lifted into position first, followed by the upper half. Then, the remaining field sections forming the north and south back spans were completed. Finally, the three field sections forming the remaining 290 feet of the 380-foot main span were bolted on barges and floated into position. Next, the two ends of the back spans (at the end piers) were jacked up by 30" above the final elevation on the permanent bearings. Temporary longitudinal movement also was permitted at one of the main pier fixed bearings so one of the back spans could be pushed back to provide adequate clearances during the lifting of the main-span segment and then brought forward into position for completing the splice. Jacking against the cantilevered ends of the main-pier girder segments then lifted the main-span section into position. After the main span splicing was complete, the girder at the end piers was lowered onto the permanent bearings, inducing the proper negative moments at the final main-span splice locations.

The temporary allowance for longitudinal movement at one of the fixed bearings was maintained until all of the dead load was affected on the bridge. This provision was needed to prevent the build-up and lock-in of very large longitudinal forces between the superstructure and the main piers that could not be economically accommodated in the foundation design.

The jacking of the 650-ton main span section was accomplished by using a total of 20 jacks with a 50-ton capacity each. The arrangement consisted of five jacks at each side of the two lifting frames.

**Deck Slab:** The 10" thick, 4500psi concrete deck slab was designed to act compositely with not only the main box girder, but also with transverse floor beams (including the cantilever out riggers) and the longitudinal fascia girders. The resulting span proportions of the desk slab leads to a fairly high level of two-way action, except for the 6-foot cantilever overhang section beyond the fascia girders. The deck is finished with a 1-1/2" LMC overlay. SIP forms were not allowed in forming the deck for the cantilever overhang or for the outer sections between the box girder and the fascia girders. DOC used removable forms supported by bar joists in these sections.

For the interior section between the webs of the box girder, SIP forms were allowed. To eliminate the increase in the additional dead load due to concrete in the 4" deep troughs of the SIP forms, Styrofoam filler was used to cover the troughs. The SIP forms were in turn supported by hanger rods extending down from temporary steel beams spanning transversely. These temporary beams were supported on a four-legged chair placed on the box girder top flanges, placing the beams about two feet above the finished deck slab.

**OWNER:** Massachusetts Turnpike Authority

**DESIGN CONSULTANT:** HNTB Corporation

**Management consultant:** Bechtel/Parsons Brinckerhoff

**General Contractor:** Daniel O'Connell's Sons, Holyoke, Massachusetts

**Steel Fabricator:** Tampa Steel Fabricators, Tampa, Florida

**Steel detailer:** Tensor Engineering, Tallahassee, Florida

**Main-span lifting:** Scanada Lift Systems Inc., Bow, New Hampshire

**Erection ENGINEERING:** Finley McNary Inc., Hartford, Connecticut

**Acknowledgements:** The authors wish to acknowledge the Massachusetts Turnpike Authority (MTA) and the Central Artery/Tunnel Project (CA/T) for their permission to publish this paper.